91602
new 82-10

ontario hydro
research division

To: Mr. M.J. Northfield
Manager
Environmental Protection
Central Production Services Branch

# LAMBTON TGS ASH STORAGE AREA STABILITY ANALYSIS

A. Vorauer

The height of the Lambton TGS ash storage area is to be increased to 18 metres above grade. A stability analysis has been undertaken to assess the feasibility of such an increase. The analysis consisted of two parts: 1) a field and laboratory testing program to evaluate the in situ ash and overburden strengths and 2) a computer-based, slope stability analysis to determine the minimum factor of safety and critical slope surface geometry. Because of the weakness of some of the foundation soils, deep-seated critical slip surfaces tend to have the smallest safety factors. The magnitude of these safety factors is highly dependent on the in situ density of the ash. The analysis also indicates that shallow slope failures will occur if the water-table within the ash is allowed to exit above the toe of the slope. Recommendations are made concerning further characterization of the in situ ash as well as water-table monitoring within the ash slopes. Interim guidelines for slope heights and slope angles are provided until such time as a more reliable stability analysis can be made.

## 1.0 INTRODUCTION

The continued production of fly ash at the Lambton Thermal Generating Station near Sarnia, Ontario, has resulted in a need to increase the on-site ash storage capacity. The present ash management method used at Lambton consists of trucking fly ash, preconditioned to 15% moisture content, to the ash disposal area where it is then end-dumped and levelled off with a front-end loader. The ash disposal area occupies 44 ha in the northeast portion of the station property and the top of the ash mound has locally reached a maximum elevation of 203 m.ASL. This represents an approximate ash height of 16 m above original grade which rises from 186.5 m.ASL along the western edge of the disposal area to 188 m.ASL along the eastern edge/1/. Lake Lambton, which occupies an area of approximately 7 ha in the northwest corner of the ash disposal area, serves as a collection pond for surface runoff. Two smaller ponds occupy the southwest corner of the disposal area as shown in Figure 1.



| job | file | date | report no. |
|---|---|---|---|
| 43-144-573 | 843.12 | January 8, 1990 | 89-314-K |

## LEGAL NOTICE

This report was prepared by Ontario Hydro and, while effort has been made to assure the accuracy and applicability of the material contained herein, neither Ontario Hydro nor any person acting on its behalf:

(a) makes any warranty, express or implied with respect to the use of any information, apparatus, method or process disclosed in this report or that such use may not infringe privately owned rights; or

(b) assumes any liabilities with respect to the use of, or for damages resulting from the use of, any information, apparatus, method or process disclosed in this report.

To increase the storage volume of the ash disposal area, it has been proposed to raise the height of the ash mound to 18 m (60 ft). The overall stability of this new ash mound has been investigated and the results are provided in the following sections.

## 2.0 FIELD INVESTIGATION

Numerous boreholes were drilled on Lambton TGS property during the initial site investigation to determine the soil and rock properties for construction of the generating station. The overburden at the site consists of several clay and clay till layers with a total thickness of 42.5 m. In general, five major soil zones have been identified/1/, in sequence from top to bottom: i) a stiff, fissured, weathered silty clay crust approximately 5 m thick, ii) a grey, silty clay with some sand and fine gravel, approximately 12.5 m thick, iii) a light grey brown clay, normally consolidated for its full depth of 10 m, iv) a dark grey silty clay with sand and medium gravel, overconsolidated for its full depth of 10 m and v) a 5 m thick basal till layer, consisting of very dense, sandy silty clay to silty sand with fine to coarse gravel sizes. The underlying bedrock in this region is known as the Port Lambton shale.

In an attempt to determine the present engineering properties of the clay underneath the ash mound, undisturbed clay samples were obtained from an augered borehole marked as location A in Figure 1. These samples were returned to the laboratory for identification and for measurements of density, moisture content and shear strength. The shear strengths were determined using a combination of unconfined and quick undrained compression tests. The results of the analyses are provided in the appendix, and the shear strengths are plotted versus depth in Figure 2.

In general, the undrained shear strength values plotted in Figure 2 agree well with those determined for a previous investigation on coal pile stability/2/. Shear strengths in this previous study were determined using field vane and dilatometer tests, as well as laboratory strength tests. The shear strength values determined during the present investigation also agree with average values calculated from a compilation of all previous data by Bennett/3/. A total of 461 laboratory-determined shear strengths, plus 465 density measurements, were compiled to determine average values for all 5 soil zones. These averages which are listed in Table I, were input for the stability analysis of the fly ash mound.

As part of a separate field investigation program, a total of four boreholes were augered into the fly ash mound (labelled A, C, D and E in Figure 1) for the purpose of collecting samples of ash for bulk chemical analysis.

Information on ash strength variability, total ash thickness and approximate water-table position was obtained from these boreholes. An indication of ash strength variability was obtained by examining the blow-counts necessary to obtain individual ash samples. As expected, the ash strength is quite variable (N-counts ranging from 0 to 95 blows/ft) due to the construction of thick individual lifts, the lack of comprehensive compaction and the interlayering of fly ash and bottom ash. Total ash thicknesses and approximate depths to water for all four boreholes are given in Table II.

Three piezocone soundings were attempted, labelled B, F and H, adjacent to boreholes A, E and C respectively. Due to the inability of the drilling rig used to adequately push the piezocone through the stiff layers, only one satisfactory sounding was obtained. The plots of tip resistance and pore pressure versus depth for location H are given in Figures 3 and 4. The plot of tip resistance versus depth confirms the presence of several stiff layers within the ash mound and indicates that the weakest zone occurs between the top of the clay, at a depth of 10.5 m, and the first stiff layer at 8 m. The pore pressure profile in Figure 4 is indicative of silt-type deposits which cause a build-up of local pore pressures during pushing of the cone, followed by dissipation to the equilibrium pressure head during rechucking. These dissipation lows, occurring at 1 metre intervals (individual rod lengths) indicate that the equilibrium water level, in the southeast corner of the ash disposal site, is at a depth of 4.75 m. This is comparable to the depth to water of 4 metres noted in adjacent borehole C.

Correlations between piezocone tip resistance and shear strength parameters for sands ($\phi$) and clays (Cu) have been developed. However, for the case of silt deposits, strength correlations are not as straightforward due to the partially-drained failure conditions generated during cone penetration. An attempt at including the magnitude of generated pore pressures into a shear strength correlation was made by Senneset and Janbu/4/ and their methodology has been applied to the cone data obtained from sounding H. The results of this analysis, while not entirely effective, indicate that $\phi$-values range from $25^{0}$ to $35^{0}$ as shown in Figure 5.

## 3.0 SLOPE STABILITY ANALYSIS

To perform the slope stability analyses for the Lambton TGS ash pile, the computer program, PC-SLOPE version 4.0, was used. This program uses the limit equilibrium method of slices to generate minimum factors of safety for specified slope morphologies and geometries. For this particular analysis, Bishop's Simplified Method was used which satisfies moment equilibrium and assumes only horizontal interslice forces.

The data input required by the program include: 1) the geometry of the slope in question, 2) the depth, thickness, unit weight and shear strength parameters of all soil layers within the possible failure zone, 3) the pore pressure conditions within the slope, and 4) slip circle parameters such as a grid of centres of rotation and a range of circle radii to be attempted. For each combination of the above parameters, a minimum factor of safety is calculated and displayed by the program.

Due to the uncertain parameters associated with the fly ash pile, such as final slope angle, water-table position, strength and unit weight of the ash, a parametric stability analyses was conducted using the aforementioned computer program. The objective of this analysis was to assess the importance of each parameter on the overall stability of the ash pile and thereby determine the most likely minimum factor of safety for the structure.

The engineering properties and thicknesses of the overburden soils used in the analysis are given in Table I. These parameters remained unchanged throughout the analysis.

A summary of the results of the parametric analysis is provided in Table III and each scenario will be discussed in detail in the following section.

### 3.1 Parametric Analysis Results

The layout of the slope stability problem studied in this analysis is given in Figure 6. Also in this figure are drawn two examples of possible failure surfaces encountered during analysis; 1) the shallow slip surface through the ash slope and 2) the deep-seated slip surface encompassing at least 25 metres of overburden.

The final ash height was fixed at 18 metres above grade and the clay berm was not included in the analysis. This berm surrounds the entire fly ash disposal area including Lake Lambton. By examining geodetic survey results and comparing to initial ground elevations, it has been determined that the height of the clay berm increases from 4.5 m along the western edge of the disposal area to 5.0 m along its eastern edge. The effect of excluding the berm from the analysis would be to produce conservative safety factors against slope failure. The resulting downward force would probably act through the centre of rotation having no effect on moment equilibrium, or to the right of centre acting as a resisting force for moment equilibrium.

Several parameters were examined during this analysis:

- Based on surface elevations derived from 1987 aerial photography, the internal and external ash slopes vary from 1.6:1 to 3.7:1. Since there is apparently little control on slope angles during past construction of the ash mound, the effect on stability of varying the slope from 2:1 to 3.5:1 was examined.

- The only water level data available for the ash mound is that given in Table II. Because of this lack of data, the effect on slope stability of varying the water level height from 15% to 50% of the slope height was examined.

- The effect of varying the engineering properties of the ash was also examined by inputting ash densities of 1280 kg/m$^3$ (80 pcf) and 1440 kg/m$^3$ (90 pcf) and $\phi$ of 25$^o$ to 30$^o$.

- The effect of having the water-table exit above the toe of the slope was studied by varying the exit height from 1 to 2 m above the toe. The importance of having the water-table exit into a pond (Lake Lambton) was also examined.

The difference between cases 1 and 2 is simply the presence of the weathered crust in the former and not in the latter. This was simulated by substituting the strength parameters for zone I with those of zone II. The effect on the minimum factor of safety is negligible, but the position of the critical slip surface has changed from a shallow failure through the ash in case I to deep-seated failure in case II (ie as shown in Figure 6).

Since the higher strength of the weathered crust is generally not mobilized because of the presence of numerous fractures, it was assumed for the remaining cases examined, that the strength of zone I was equivalent to that of zone II. This procedure caused deep-seated failures to dominate the analysis results.

Cases 2, 3 and 4 represent the scenarios of three different water level heights within the ash slope. This parameter has no effect on the minimum factor of safety or on the position of the critical slip surface which continues to be deep-seated.

Cases 5, 6 and 7 represent slope geometries with decreasing slope angles (2.5:1, 3.0:1 and 3.5:1). As expected, the effect of decreasing the slope angle is to increase the minimum factor of safety. However, this increase is only marginal since the critical slip surface continues to be deep-seated and the decrease in the ash slope angle only marginally decreases the weight acting on the overburden.

It is assumed that an acceptable factor of safety (FS) against slope failure is 1.3. This FS is only approached for case 7 where the slope angle was set at 3.5 (horizontal) to 1 (vertical) and the slope height is 18 metres above grade. This case also assumed that the ash density was equal to 1440 kg/m$^3$ (90 pcf) which is a value associated with well compacted ash. For case 8, the slope angle was increased to 2.5:1 and the ash density was decreased to 1280 kg/m$^3$ (80 pcf). As shown in Table III, the FS for this case increased to an acceptable 1.332 even though the slope angle was increased by $6^0$ compared to case 7. The increase in FS due to the reduction in density alone is determined by comparing case 5 (FS = 1.195) to case 8 (FS = 1.332). This comparison illustrates the importance of having a reasonably accurate knowledge of the in situ ash density for the purposes of stability calculations at Lambton TGS. At this time, a probable upper bound of 1440 kg/m$^3$ has been used for conservative analysis.

Reduction of the effective shearing resistance of fly ash will have little effect on the minimum factor of safety of the mound unless it causes the FS to go below the value associated with deep-seated failure. In such a case, the critical slip surface would be of the shallow type, cutting through the ash slope. If a plane translational slip failure is assumed for this case, a simple relationship can be adopted to assess the factor of safety. This type of failure can be assumed if the failure surface is parallel to the surface of the slope and its depth is small compared to the length of the slope. If the soil between the surface and the failure plane is not fully saturated, then/5/:

$$FS = \frac{\tan \phi'}{\tan \beta}$$

where, $\phi'$ = effective angle of shearing resistance of the material in question (degree)
$\beta$ = slope angle (degree)

If $\phi' = 30^0$ and $\beta = 26.5^0$ (2:1), the factor of safety is 1.158 which is approximately equal to the FS for case 1 associated with a shallow critical slip surface. If $\phi' = 25^0$ and $\beta = 21.8^0$, $18.4^0$ and $15.9^0$, as in cases 5 to 7, the corresponding safety factors are 1.166, 1.402, 1.637 respectively. This simple analysis indicates that only in the case of a slope angle approaching $21.8^0$ (2.5:1) will the critical slip surface go through the ash slope. The FS will only equal 1.0 (ie failure) for the case of a slope of 2.1 (horizontal) to 1.0 (vertical) (ie $\beta = 25^0$) or steeper.

A significant parameter which will influence the likelihood of shallow failures, is whether the water-table within the ash mound exits above the toe of the slope. This case is illustrated by cases 9, 10 and 11 in Table III. If the water-table exits 1.5 metres above the toe of slope (case 10), the FS reduces to 1.117 as compared to 1.195 for case 5 where water exits 1.0 m above the toe. The critical slip surface also passes entirely through the ash in case 10. If the water-table exits 2 m above the toe (case 11), a shallow slope failure is assured (ie FS < 1.0).

Cases 12, 13 and 14 deal with the scenario of a water-table exiting above the toe, but into a reservoir of water (Lake Lambton). The depth of the reservoir was assumed to be 1 metre below the toe of the slope. The width of the reservoir was assumed to be at least 150 metres. For both cases 12 and 13, the critical slip surfaces remain deep-seated with factors of safety approximately 1.119. For case 14, the reduced ash density caused the critical slip surface to pass through the ash slope, but the minimum factor of safety was marginally better than case 10 due to the presence of the weight of water as a resisting force.

One final scenario was examined for a reduced slope height of 13 metres (elevation of 199.5 to 201 m ASL), a slope angle of 2.5 to 1 and $\phi' = 30^0$. The critical slip surface is through the ash slope with a minimum factor of safety of 1.449. This is in contrast to case 5 which resulted in a deep-seated failure surface with FS = 1.195 for a slope height of 18 metres.

## 4.0 CONCLUSIONS

Based on the results of one borehole (A), the undrained shear strengths of the overburden under the ash pile are similar to those obtained in the past.

Piezocone tip resistance and blow-count profiles through the ash identify the presence of several stiff layers interbedded with weaker material. The weakest ash layer is approximately 2 metres thick, located at the bottom of the ash pile. Tentative shear strength correlations based on piezocone data indicate that the angle of shearing resistance ($\phi$) for the ash ranges from $25^0$ to $35^0$.

Based on the results of the parametric slope stability analyses, the critical slip surface for an 18 m high ash mound is deep-seated, reaching a depth of 25 m below grade. This result is similar to the one obtained from past stability analyses for a gypsum stack located on Lambton TGS property/3/. Decreasing the angle of the ash slope from 2:1 to 3.5:1 increases the factor of safety from 1.159 to 1.262. These factors can be further increased by assuming lower densities for the ash. This is because a lower ash density decreases the

driving force for deep-seated failures. However, since no measurements of in situ ash density are available, the higher density (1440 kg/m$^3$) should be assumed for the time being.

Critical slip surfaces passing only through the ash slope could be obtained in three cases: 1) assuming a higher strength, weathered crust just below the ash, 2) forcing the water-table to exit more than 1.5 m above the toe of the slope, 3) assuming $\phi' = 25^0$ for the ash and making the slope 2.5:1. In all three cases, the factors of safety are less than 1.2 which is unacceptable for this type of structure. A $\phi'$-angle of $25^0$ is probably low since slopes of more than 2.1 to 1 exist at several locations witnin the ash disposal area.

## 5.0 RECOMMENDATIONS

Until a more accurate estimate of in situ ash density can be made, an ash mound reaching 18 m in height above grade (204.5 to 206 m.ASL from west to east) should have side slopes of 3.5 to 1 or less. If this criteria cannot be met, the ash mound should be limited to 17 m with side slopes of 2.5 to 1 (FS = 1.26). A program of measuring in situ ash densities could be incorporated into the annual coal pile density testing for inventory purposes (ie March 1990). A more accurate estimate of the height of water within the ash mound could be obtained by installing piezometers concurrent with the density tests. Water levels under the critical ash slopes should also be monitored to ensure that they do not exit above the toe of the slopes, which may result in slumping failures.

Since the elevation of the fly ash mound is approaching the "critical" level, it is recommended that periodic elevation checks be done to ensure that recommended levels are not exceeded.

Approved by:

T.W. Klym
Manager
Civil Research Department

Prepared by:

A. Vorauer
Engineer
Soil Sciences Section

Reviewed by:

C.B.H. Cragg
Senior Engineer
Soil Sciences Section

AV:LM

## REFERENCES


1. Hanna, T.H. "Engineering Properties of the Overburden Soils at Lambton GS Site". Ontario Hydro Research Division Report No 66-186-K, 1966.

2. Cragg, C.B.H. "Review of Coal Stockpile Stability at Lambton TGS". Research Division Report No 82-427-K, 1982.

3. Bennett, A.R. "Lambton GS Gypsum Stack Stability Assessment - Probabilistic Approach". Geotechnical Engineering Department, 1982.

4. Senneset, K. and Janbu, N. "Shear Strength Parameters Obtained from Static Cone Penetration Tests". Strength Testing of Marine Sediments: Laboratory and In Situ Measurements, ASTM STP 883, Philadelphia, 1985, pp 41-54.

5. Craig, R.F. "Soil Mechanics". 2nd Ed, Van Nostrand Reinhold Company, New York, 318p.

TABLE I

Overburden Properties Used for Analysis

| Soil Type | Unit Weight ($kN/m^3$) | Cu kPa | Thickness (m) |
|---|---|---|---|
| Zone I | 19.2 | 170 | 5.0 |
| Zone II | 18.6 | 59 | 12.5 |
| Zone III | 17.7 | 48 | 10.0 |
| Zone IV | 19.7 | 69 | 10.0 |

TABLE II

Ash Mound Properties from Borehole Logs

| Borehole | Ash Thickness (m) | Depth to Water (m) | Water-Table Elevation (m.ASL) | Range of N-Count |
|---|---|---|---|---|
| A | 9.6 | - | - | 0 - 30 |
| C | 10.4 | 4.0 | 194.6 | 0 - 21 |
| D | 16.7 | 12.2 | 190.2 | 2 - 30 |
| E | 16.7 | 12.8 | 190.2 | 4 - 95 |

TABLE III

Results of Parameter Stability Analysis

Slope Height = 18 m

| Case Number | Slope Angle | Height of Water Slope Height | Ash Parameters $\rho$ (kg/m$^3$), $\phi'$ | Water Exit Height Above Toe (m) | Minimum Factor of Safety | Depth of Failure Arc (m below grade) | Comments |
|---|---|---|---|---|---|---|---|
| 1 | 2:1 | 0.50 | 1440, 30$^o$ | - | 1.163 | - | - weathered crust as Zone I |
| 2 | 2:1 | 0.50 | 1440, 30$^o$ | - | 1.159 | 25 | - no crust |
| 3 | 2:1 | 0.25 | 1440, 30$^o$ | - | 1.159 | 25 | - deep seated failure |
| 4 | 2:1 | 0.15 | 1440, 30$^o$ | - | 1.159 | 25 | |
| 5 | 2.5:1 | 0.50 | 1440, 30$^o$ | - | 1.195 | 25 | |
| 6 | 3.0:1 | 0.50 | 1440, 30$^o$ | - | 1.216 | 25 | |
| 7 | 3.5:1 | 0.50 | 1440, 30$^o$ | - | 1.262 | 25 | |
| 8 | 2.5:1 | 0.50 | 1260, 30$^o$ | - | 1.332 | 0 | - shallow failure |
| 9 | 2.5:1 | 0.50 | 1440, 30$^o$ | 1.0 | 1.192 | 25 | |
| 10 | 2.5:1 | 0.50 | 1440, 30$^o$ | 1.5 | 1.117 | 0 | |
| 11 | 2.5:1 | 0.50 | 1440, 30$^o$ | 2.0 | 0.956 | 0 | |
| 12 | 2.5:1 | 0.50 | 1440, 30$^o$ | 1.0* | 1.183 | 25 | - water exits to pond |
| 13 | 2.5:1 | 0.50 | 1440, 30$^o$ | 1.5* | 1.199 | 25 | |
| 14 | 2.5:1 | 0.50 | 1260, 30$^o$ | 1.5* | 1.222 | 0 | |
| 15 | 2.5:1 | 0.50 | 1440, 30$^o$ | - | 1.449 | -5 | - slope height = 13 m |
| 16 | 2.5:1 | 0.50 | 1440, 30$^o$ | - | 1.327 | 25 | - slope height = 16 m |

*Water-table exits into a pond (Lake Lambton)

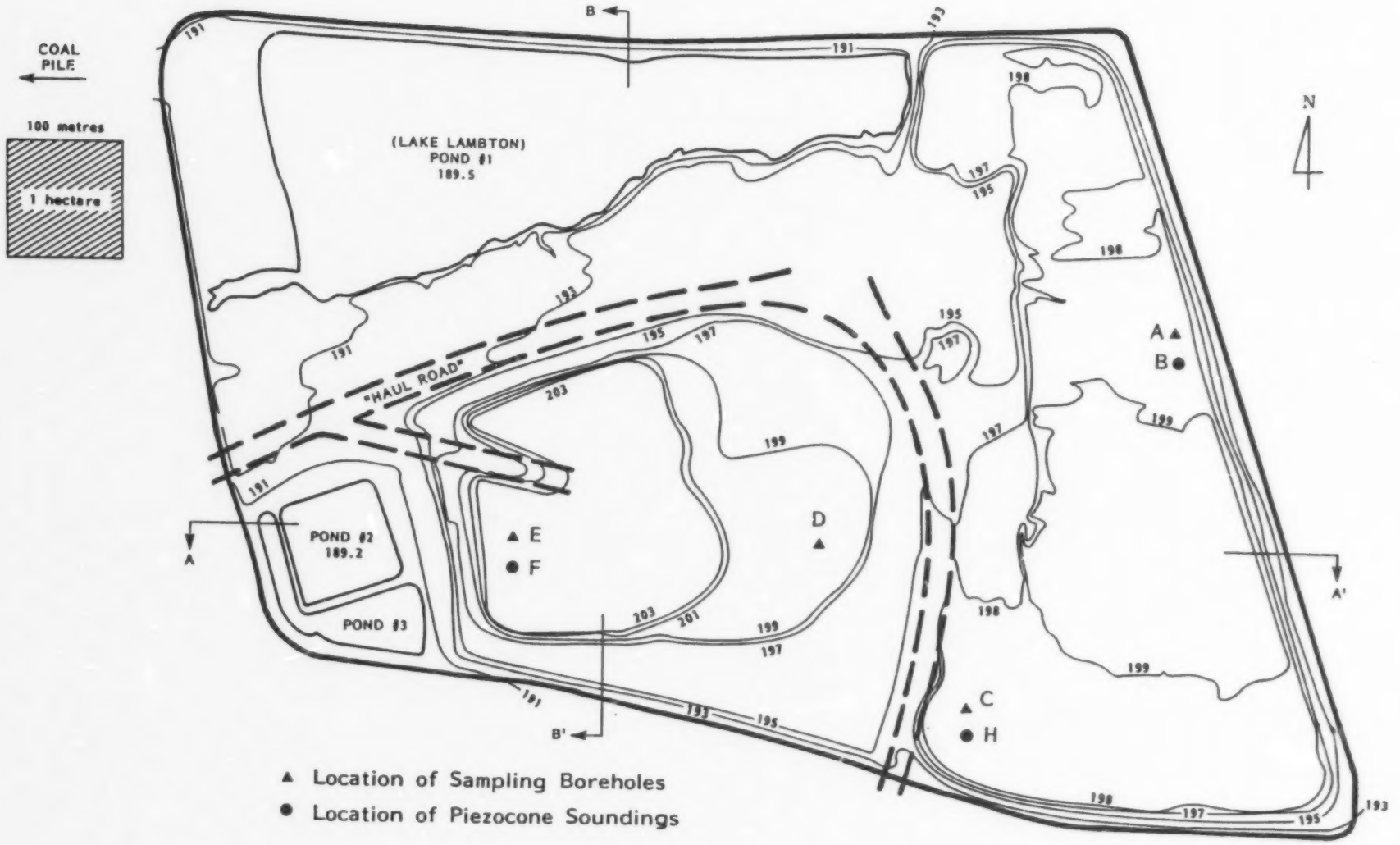


FIGURE 1
ASH STORAGE AREA CONTOUR PLAN 1987



89-314
200914-RD

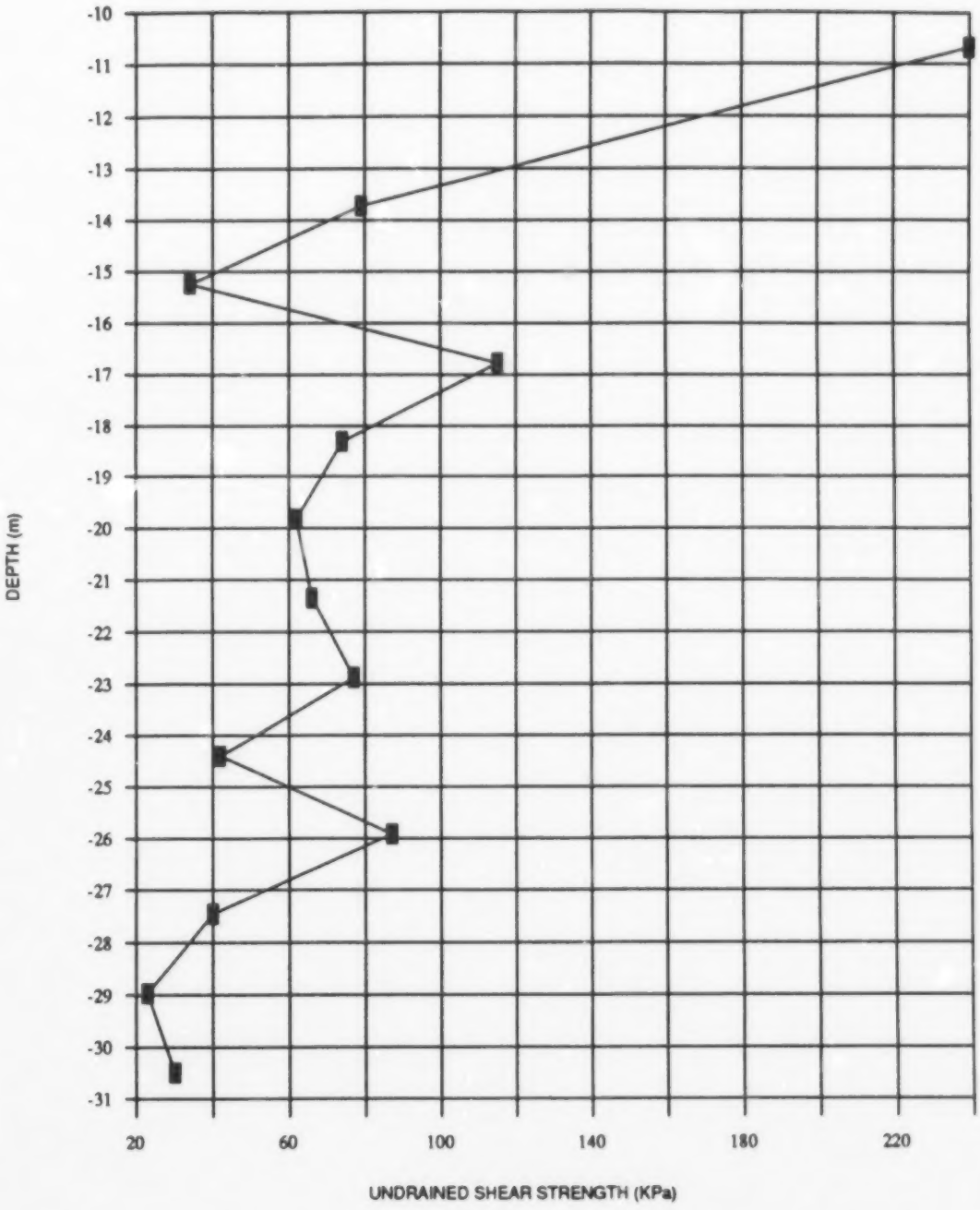


FIGURE 2
UNDRAINED SHEAR STRENGTH PROFILE FOR OVERBURDEN
UNDER LAMBTON TGS ASH PILE (BOREHOLE A)

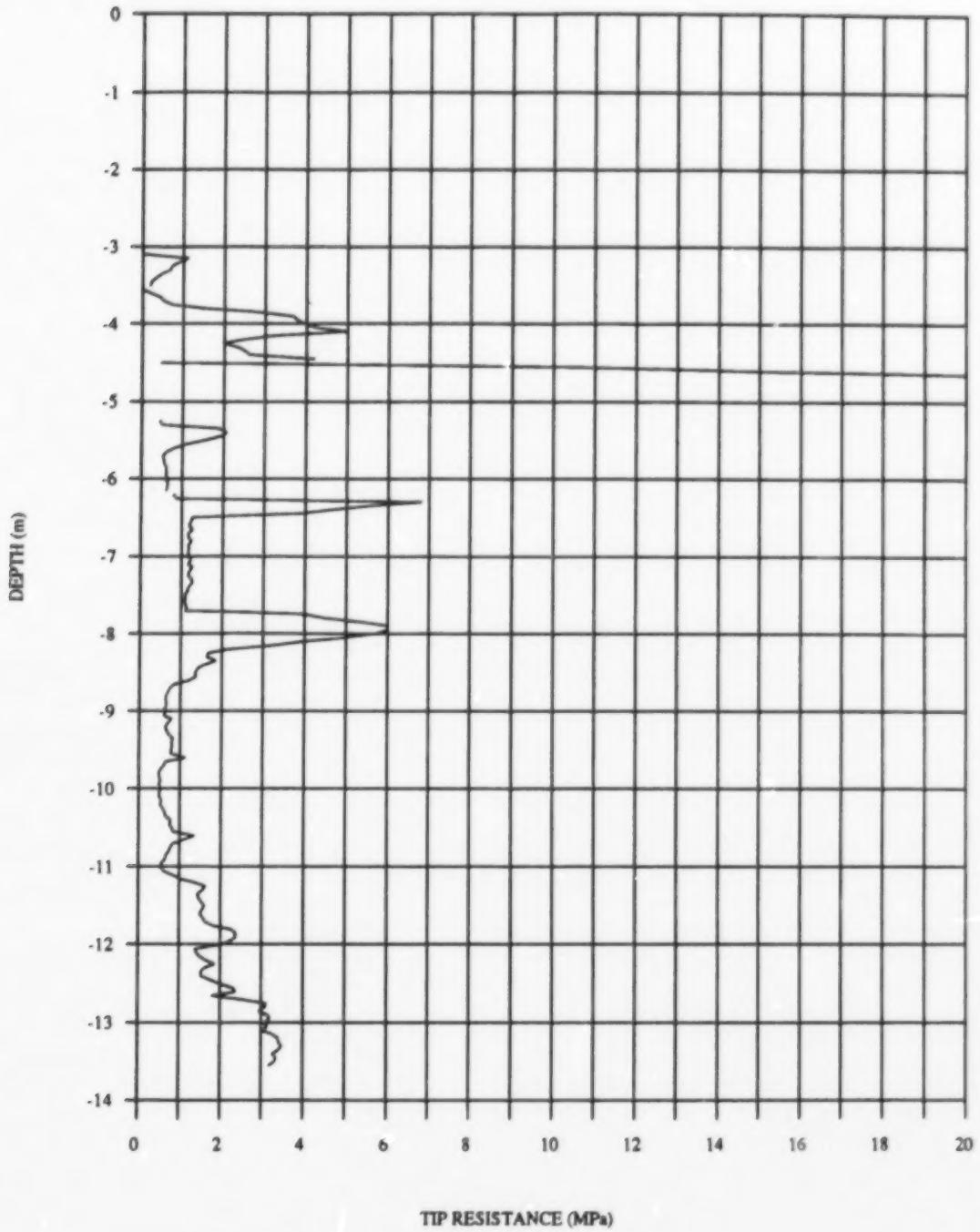


FIGURE 3
LAMBTON TGS ASH PILE
TIP RESISTANCE PROFILE FROM PIEZOCONE SOUNDING H

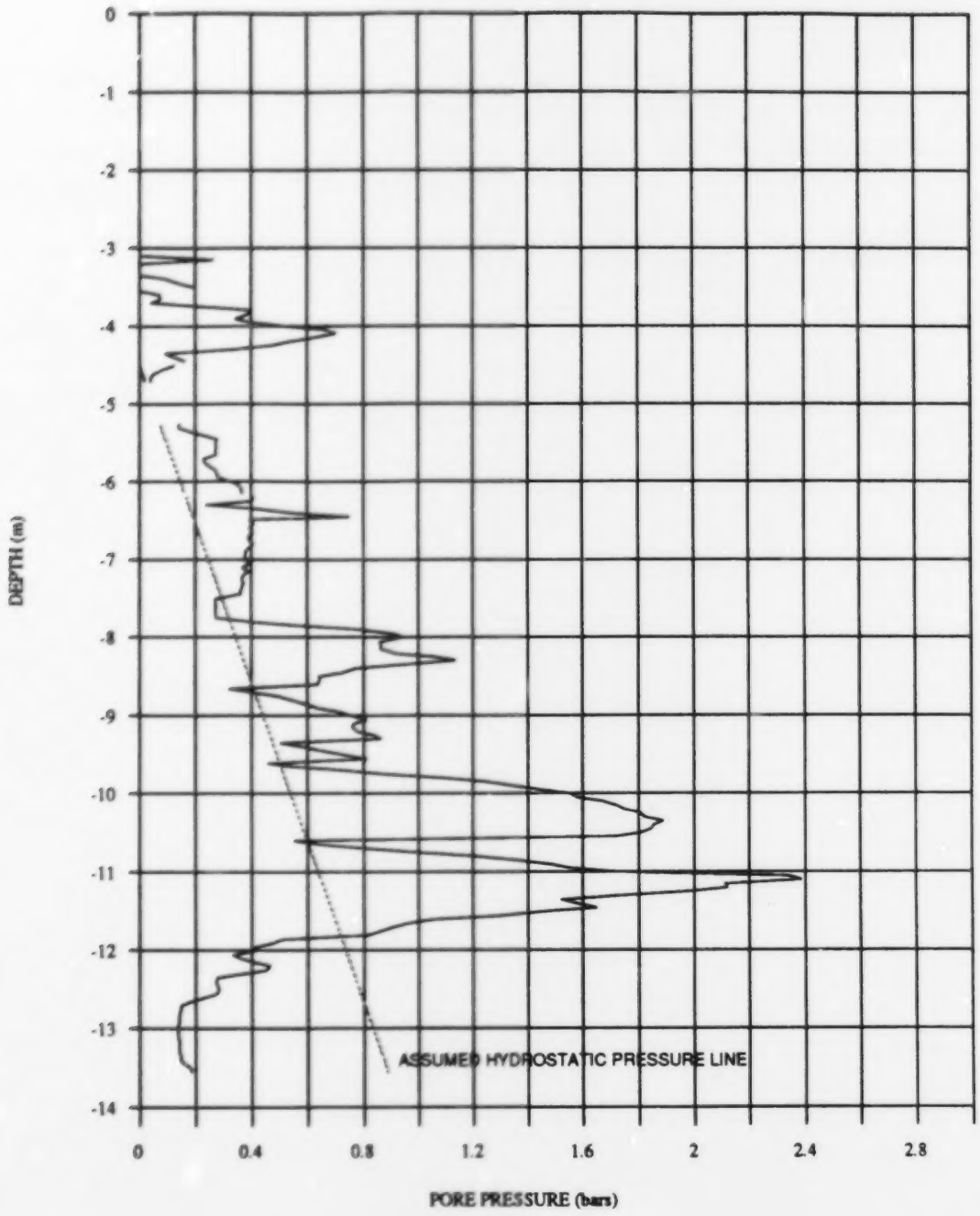


FIGURE 4
LAMBTON TGS ASH PILE
PORE PRESSURE PROFILE FROM PIEZOCONE SOUNDING H

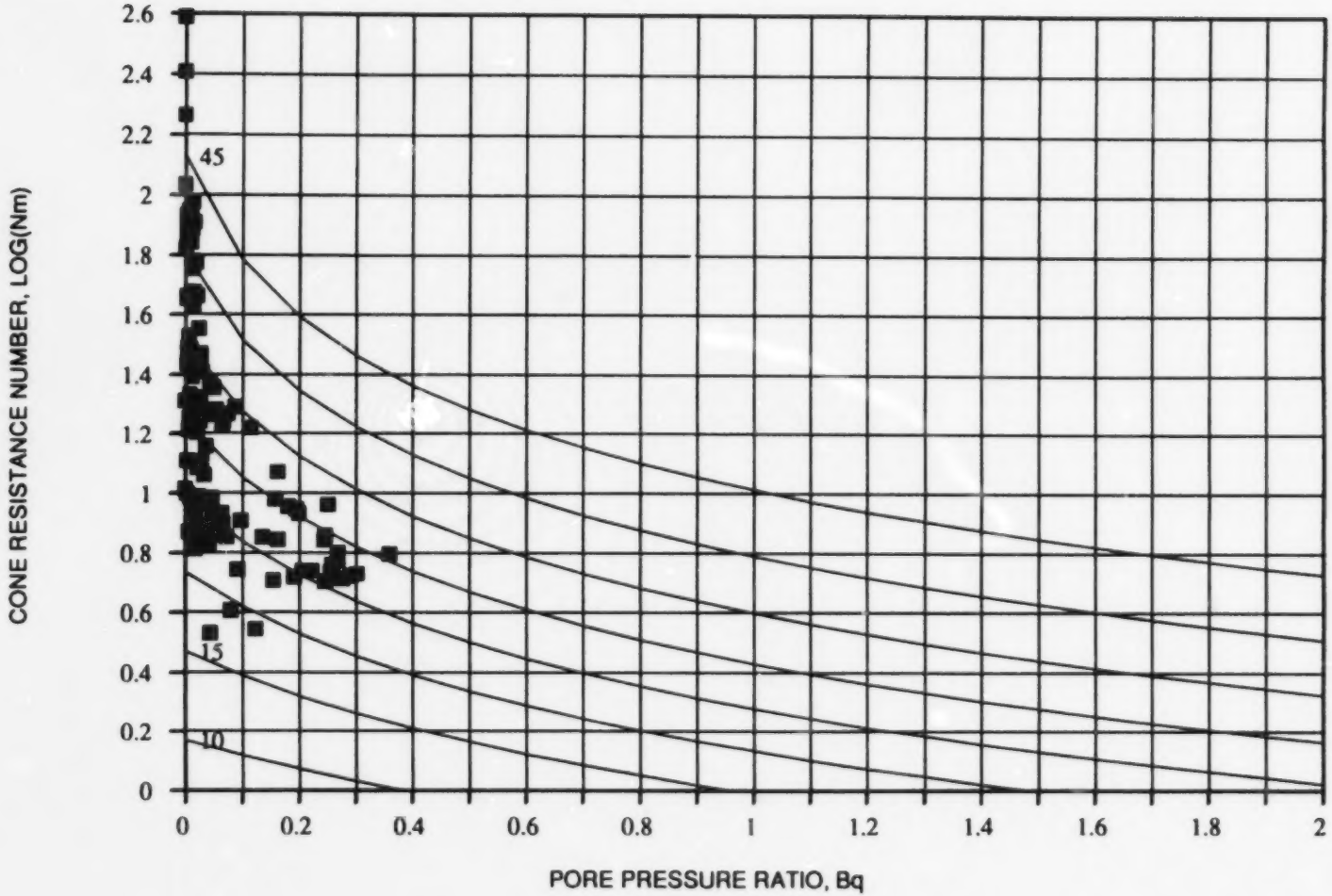


FIGURE 5
Ø ESTIMATES FOR LAMBTON ASH BASED ON PIEZOCONE DATA
METHOD OF SENNESET & JANBU (1985)

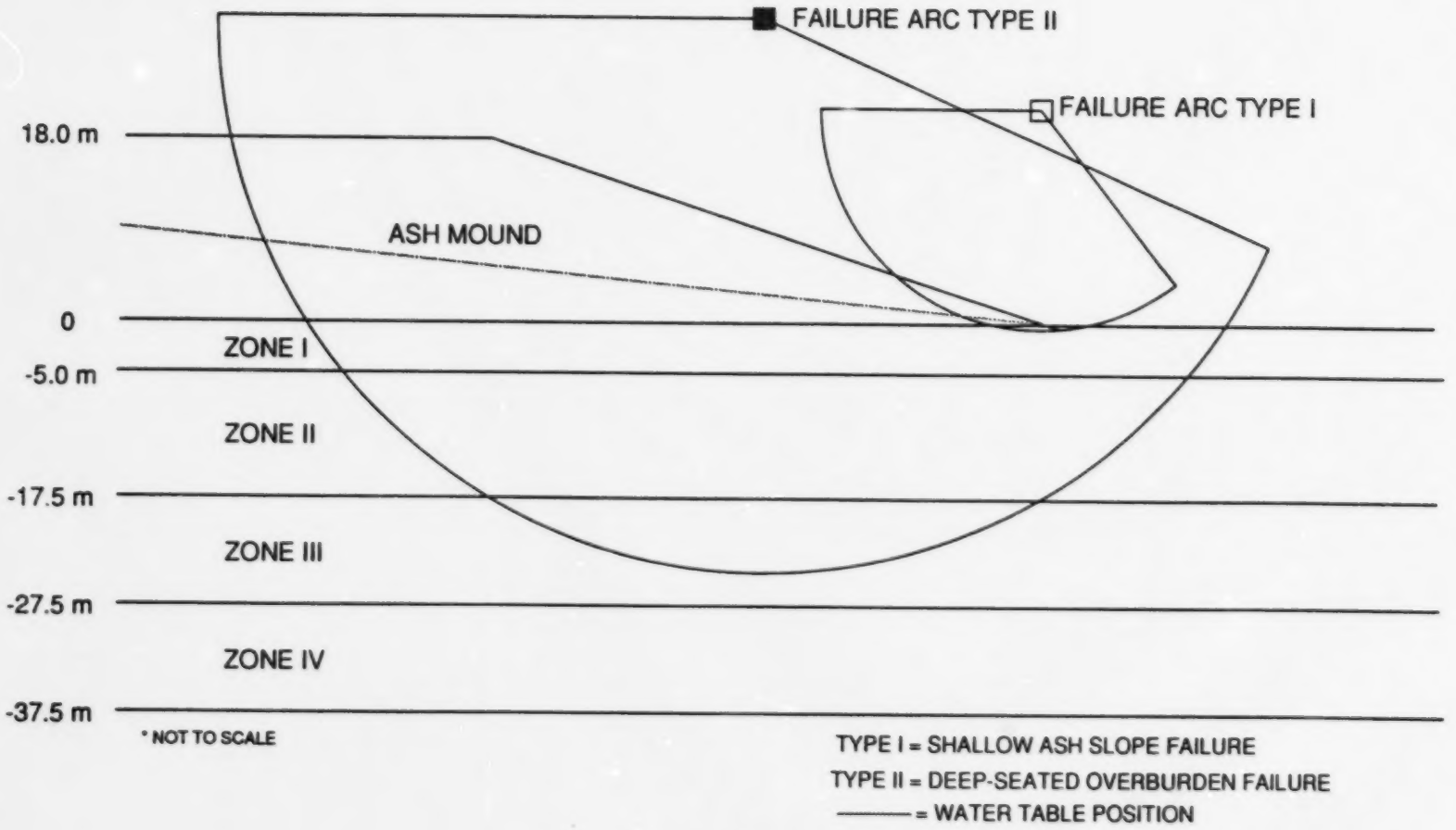


FIGURE 6
LAMBTON TGS ASH PILE
SCHEMATIC OF TWO POSSIBLE CRITICAL SLIP SURFACES

# APPENDIX

ONTARIO HYDRO
RESEARCH DIVISION

| HOLE No. | PROJECT — | REMARKS |
|---|---|---|
| BH-A | LAMBTON T. G. S. ASH | |

| SAMP. No. | LABORATORY SOIL CLASSIFICATION | NAT. DENS. $kg/m^3$ (x.062 =PCF) | NAT. M.C. % DRY WT. | SHEAR STRENGTH kPa (x20.89 = PSF) U/C | Q/U | C/U | SHEAR STRENGTH PARAMETERS C' kPa | Φ' DEG. | PART. SIZE ANAL. | PROCTOR OPT. M/C | CONSOL. | PERM. cm/sec at 20 C° | ATTERBERG LIMITS L. L. | P. L. | P. I. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7 | DEPTH - ft 35.0 36.5 — 50 mm SHELBY — Hard, Damp, Brown, | 1913 | 20.5 | 240 | | | | | | | | | | | |
| | weathered Silt with some clay and some fine sand. Top 2" was coal ash. | | | | | | | | | | | | | | |
| 8 | DEPTH - ft 45.0 46.3 — 50 mm SHELBY — Stiff, Damp, Grey | 1944 | 28.2 | 79 | | | | | | | | | | | |
| | Clay with some silt and a trace of coarse sand. | | | | | | | | | | | | | | |
| 9 | DEPTH - ft 50.0 51.5 — 50 mm SHELBY — Firm, Damp, Grey | 1906 | 28.7 | 34 | 38 | | | | | | | | | | |
| | silty Clay. | | | | | | | | | | | | | | |
| 10 | DEPTH - ft 55.0 56.7 — 50 mm SHELBY — Very Stiff, Damp, Grey, | 1919 | 26.5 | 115 | | | | | | | | | | | |
| | silty Clay. | | | | | | | | | | | | | | |
| 11 | DEPTH - ft 60.0 61.5 — 50 mm SHELBY — Stiff, Damp, Grey, | 1943 | 25.8 | 74 | | | | | | | | | | | |
| | silty Clay with a trace of coarse sand. | | | | | | | | | | | | | | |

ONTARIO HYDRO
RESEARCH DIVISION



| HOLE No. | PROJECT — | REMARKS |
|---|---|---|
| BH-A | LAMBTON T. G. S. ASH | |

| SAMP. No. | LABORATORY SOIL CLASSIFICATION | NAT. DENS. kg/m³ (x.062 =PCF) | NAT. M.C. % DRY WT. | SHEAR STRENGTH kPa (x20.89 = PSF) | | | SHEAR STRENGTH PARAMETERS | | PART. SIZE ANAL. | PROCTOR OPT. M/C | CONSOL. | PERM. cm/sec at 20 C° | ATTERBERG LIMITS | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | U/C | Q/U | C/U | C' kPa | Φ° DEG. | | | | | L. L. | P. L. | P. I. |
| 12 | DEPTH - ft 65.0 66.5 50 mm SHELBY Stiff, Damp, Grey, | 1887 | 26.9 | 62 | | | | | | | | | | | |
| | silty Clay. | | | | | | | | | | | | | | |
| 13 | DEPTH - ft 70.0 71.5 50 mm SHELBY Stiff, Damp, Grey | 1919 | 26.5 | 66 | | | | | | | | | | | |
| | silty Clay with a trace of coarse sand. | | | | | | | | | | | | | | |
| 14 | DEPTH - ft 75.0 76.5 50 mm SHELBY Stiff, Damp, Grey | 1959 | 27.0 | 77 | | | | | | | | | | | |
| | silty Clay with a little medium to coarse sand. | | | | | | | | | | | | | | |
| 15 | DEPTH - ft 80.0 81.5 50 mm SHELBY Firm, Damp, Grey, | 1911 | 27.0 | 42 | 32 | | | | | | | | | | |
| | silty Clay with a trace of medium to coarse sand. | | | | | | | | | | | | | | |
| 16 | DEPTH - ft 85.0 86.5 50 mm SHELBY Stiff, Damp, Grey, | 1924 | 27.2 | 87 | | | | | | | | | | | |
| | silty Clay with a trace of medium sand. | | | | | | | | | | | | | | |

ONTARIO HYDRO
RESEARCH DIVISION

| HOLE No. | PROJECT — | REMARKS |
|---|---|---|
| BH-A | LAMBTON T. G. S. ASH | |

| SAMP. No. | LABORATORY SOIL CLASSIFICATION | NAT. DENS. kg/m³ (x.062 =PCF) | NAT. M.C. % DRY WT. | SHEAR STRENGTH kPa (x20.89 = PSF) U/C | Q/U | C/U | SHEAR STRENGTH PARAMETERS C' kPa | Φ' DEG. | PART. SIZE ANAL. | PROCTOR OPT. M/C | CONSOL. | PERM. cm/sec at 20 C° | ATTERBERG LIMITS L. L. | P. L. | P. I. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 17 | DEPTH - ft 90.0 91.6 — 50 mm SHELBY — Firm, Damp, Grey, silty Clay. | 1912 | 26.9 | 40 | 37 | | | | | | | | | | |
| 18 | DEPTH - ft 90.0 96.5 — 50 mm SHELBY — Soft, Damp, Grey silty Clay. | 1922 | 29.8 | | 23 | | | | | | | | | | |
| 19 | DEPTH - ft 100.0 101.3 — 50 mm SHELBY — Firm, Damp, Grey Clay with some silt. | 1892 | 29.9 | | 30 | | | | | | | | | | |
| | DEPTH - ft | | | | | | | | | | | | | | |
| | DEPTH - ft | | | | | | | | | | | | | | |

20

89-314